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Slope Stability Assessments

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This report documents the slope stability evaluations that were performed to aid in the design of the liner system for the INEEL CERCLA Disposal Facility (ICDF) landfill and ICDF evaporation pond. These stability evaluations included veneer stability, global stability, and stability after excavation. Veneer stability involves evaluation of the potential for sliding of the drainage layer on the liner system before refuse is placed. Global stability involves evaluation of the potential for sliding during operation of the landfill and of the stability of the final landfill configuration with the cover in place. Stability after excavation involves evaluation of stability, immediately after excavation of the landfill and before placement of the lining system. Veneer and global stability of the final cover configuration were also evaluated.

Results of stability analyses were used to assess the adequacy of the design configuration for the landfill and evaporation pond. Guidance for waste placement operations and practical construction and maintenance considerations are also included in this report, based on these evaluations.

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ABSTRACT

This report documents the slope stability evaluations that were performed to aid in the design of the liner system for the INEEL CERCLA Disposal Facility landfill and INEEL CERCLA Disposal Facility evaporation pond. These stability evaluations included veneer stability, global stability, and stability after excavation.

Veneer stability involves evaluation of the potential for sliding of the drainage layer and operations layer on the liner system before refuse is placed. Global stability involves evaluation of the potential for sliding during operation of the landfill, for stability of the final landfill configuration with the cover in place, and for global failure of the evaporation ponds. Stability after excavation involves evaluation of stability, immediately after excavation of the landfill and before placement of the lining system. Veneer and global stability of the final cover configuration were also evaluated.

Results of stability analyses were used to assess the adequacy of the design configuration for the landfill and evaporation pond. Guidance for waste placement operations and practical construction and maintenance considerations are also included in this report, based on these evaluations.

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ACRONYMS

ASTM	American Society for Testing Materials
CCL	compacted clay liner
CDN	composite drainage net (geocomposite)
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CETCO	Colloid Environmental Technologies Company American
CHRL	Cedar Hills Regional Landfill
DOE-ID	Department of Energy Idaho Operations Office
EPA	Environmental Protection Agency
FHWA	Federal Highway Administration
FS	factor of safety
g	acceleration due to gravity
GCL	geosynthetic clay liner
HDPE	high-density polyethylene
HELP	Hydraulic Evaluation of Landfill Performance
ICDF	INEEL CERCLA Disposal Facility
INEEL	Idaho National Engineering and Environmental Laboratory

Slope Stability Assessments

1. INTRODUCTION

Slope stability evaluations were performed to aid in the design of the liner system for the INEEL CERCLA Disposal Facility (ICDF) landfill and evaporation pond. Stability evaluations for this project were divided into three categories, namely: (a) stability after excavation; (b) veneer stability; and (c) global stability. The first category involves evaluation of stability immediately after excavation of the landfill but before placement of the lining system. Veneer stability involves evaluation of the potential for sliding of the drainage and operations layers on the liner system before refuse is placed. Global stability involves evaluation of the potential for sliding after refuse is placed and after placement of the final cover (i.e., the final landfill configuration). In the global stability analysis, the refuse mass can potentially slide on a plane through the refuse, on the lining system, or on some combination of the two. The following sections describe the methods of analysis, the cases analyzed, input parameters and assumptions, and the results of analyses.

The slope stability computer program PC-STABL5 (FHWA 1988) was used to analyze the factor of safety (FS) for the stability after excavation and for the global stability. This computer program uses a two-dimensional limiting equilibrium method for the general solution of slope stability. The program can implement either the method of slices (e.g., Bishop) or sliding block (e.g., Janbu) procedures to calculate the FS of the slope.

For the global stability analysis, earthquake loading was modeled in PC-STABL5 using a pseudo-static method. This procedure is similar to a static slope stability analysis except that the effect of earthquake loading is added as a horizontal inertial force acting at the centroid of the critical sliding mass. This inertial force is computed from the mass of the sliding volume and the seismic coefficient, k, which was defined as one-half of the peak bedrock acceleration divided by the acceleration due to gravity (g). This definition of k allows the transient pulsating nature of the earthquake to be represented as an equivalent horizontal load that is applied continuously and in one direction only. Experience has been that if the FS under the simulated earthquake loading is equal to or greater than 1.0, displacement of the slope will be less than 3 ft (Hynes and Franklin 1984; EPA 1994).

For veneer stability, the FSs were evaluated using the spreadsheet program SLOPBASE. The SLOPBASE program uses the calculation methods presented in Druschel and Underwood (1993). This program expands the traditional sliding block analysis by including anchorage forces, seepage forces, equipment loads, and the effect of toe buttressing. The sliding block analysis is a traditional geotechnical technique in which all the forces acting on the sliding block are summed and the resultant must equal zero for the block to be stable. Sliding block analysis is similar to infinite slope analysis, another traditional geotechnical method, but includes the contribution of the slope change at the bottom of the slope. FSs for sliding block analyses are developed by evaluation of the shear strengths required to balance forces and achieve stability. Earthquake loading is treated by the program as a pseudo-static force (equal to the soil weight and multiplied by a seismic coefficient, k) acting parallel to the slope.

Results of stability analyses are expressed in terms of FSs. These FSs are then compared to the minimum required values, which are either based on minimum technical guidance (EPA 1994) or the standard of practice (Sharma and Lewis 1994; Abramson et al. 1998). Based on the guidance references stated above, the following minimum FSs are used in this analysis for the specified loading conditions:

Static Loading:

- Long-Term: FS = 1.5
- Short-Term: FS = 1.3.

Seismic Loading:

- Long-Term: FS = 1.3
- Short-Term: FS = 1.1.

The analyses and assumptions presented in this report were based on the results of the subsurface investigation conducted at the project site during the conceptual design. Results of this investigation are summarized in a geotechnical report prepared for this project (DOE-ID 2000).

2. STABILITY AFTER EXCAVATION

This analysis covers the stability of the landfill slope immediately after excavation and before placement of the lining system. Because of the temporary nature of the excavation, only static loading was considered in the analysis. Both dry and partially saturated conditions were investigated to establish the range of possible conditions that could occur. In the partially saturated case, groundwater was assumed to rise to an elevation of up to 5 ft below the cell excavation. This groundwater location is considered to be very conservative, as geotechnical information for the site suggests that groundwater does not reach this height (DOE-ID 2000). A soil friction angle of 38 degrees was used for the granular native material. This value was based on the subsurface information and shear strength test data provided in the geotechnical report (DOE-ID 2000) and supplemented by engineering judgment and experience with similar soil deposits.

Stability analyses were conducted using the program PCSTABL5 (FHWA 1988). Results of these analyses are plotted in Figures 2-1 and 2-2. These results indicate that the FS of the steepest slope (2H:1V) immediately after excavation is at least 1.6, even if groundwater is considered to rise to about 5 ft below the cell excavation. This FS is higher than the required minimum for slopes and embankments (Abramson et al. 1998). Therefore, the landfill slope should be stable immediately after excavation and before placement of the lining system.

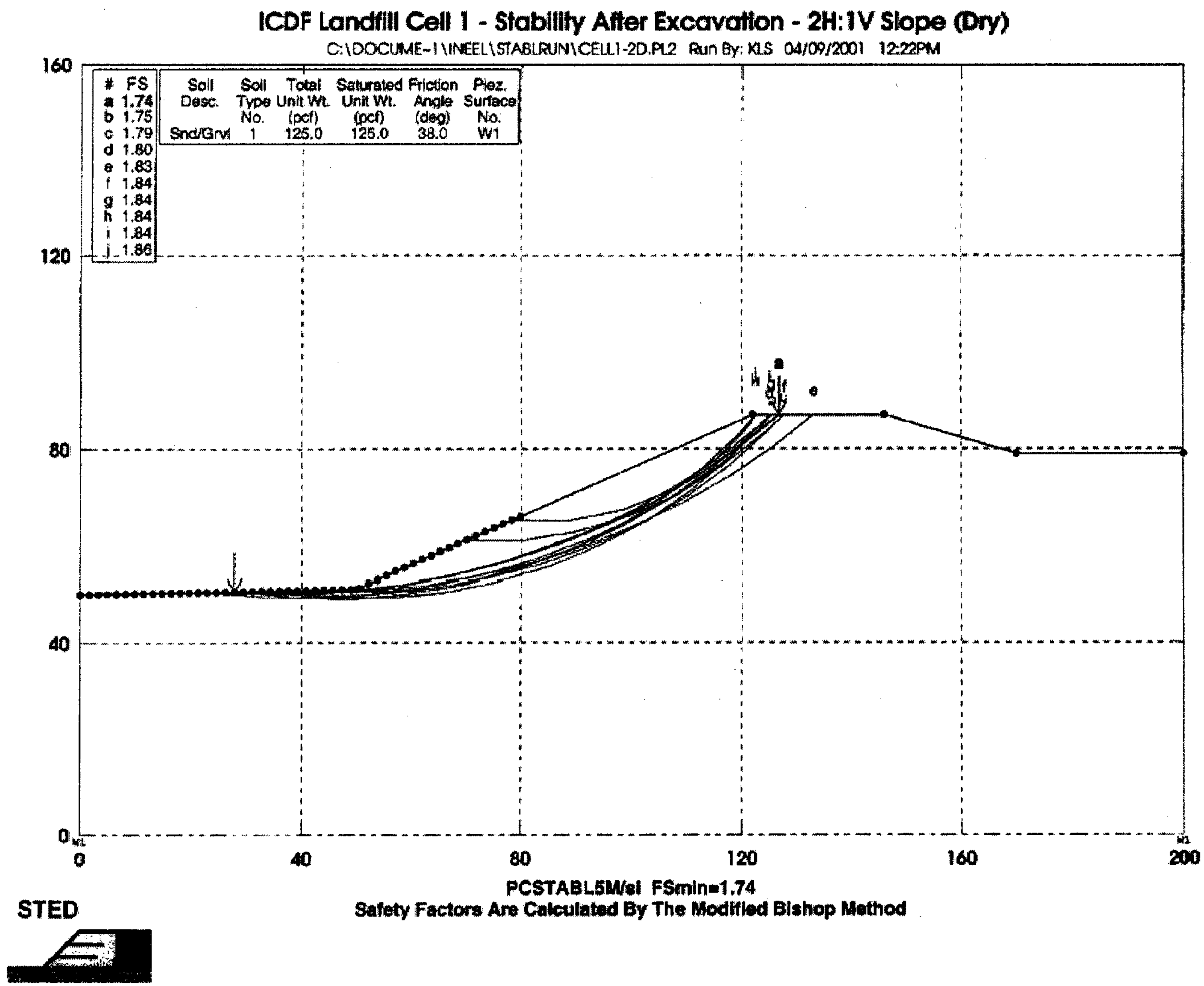

Figure 2-1. Plot of stability after excavation (dry).

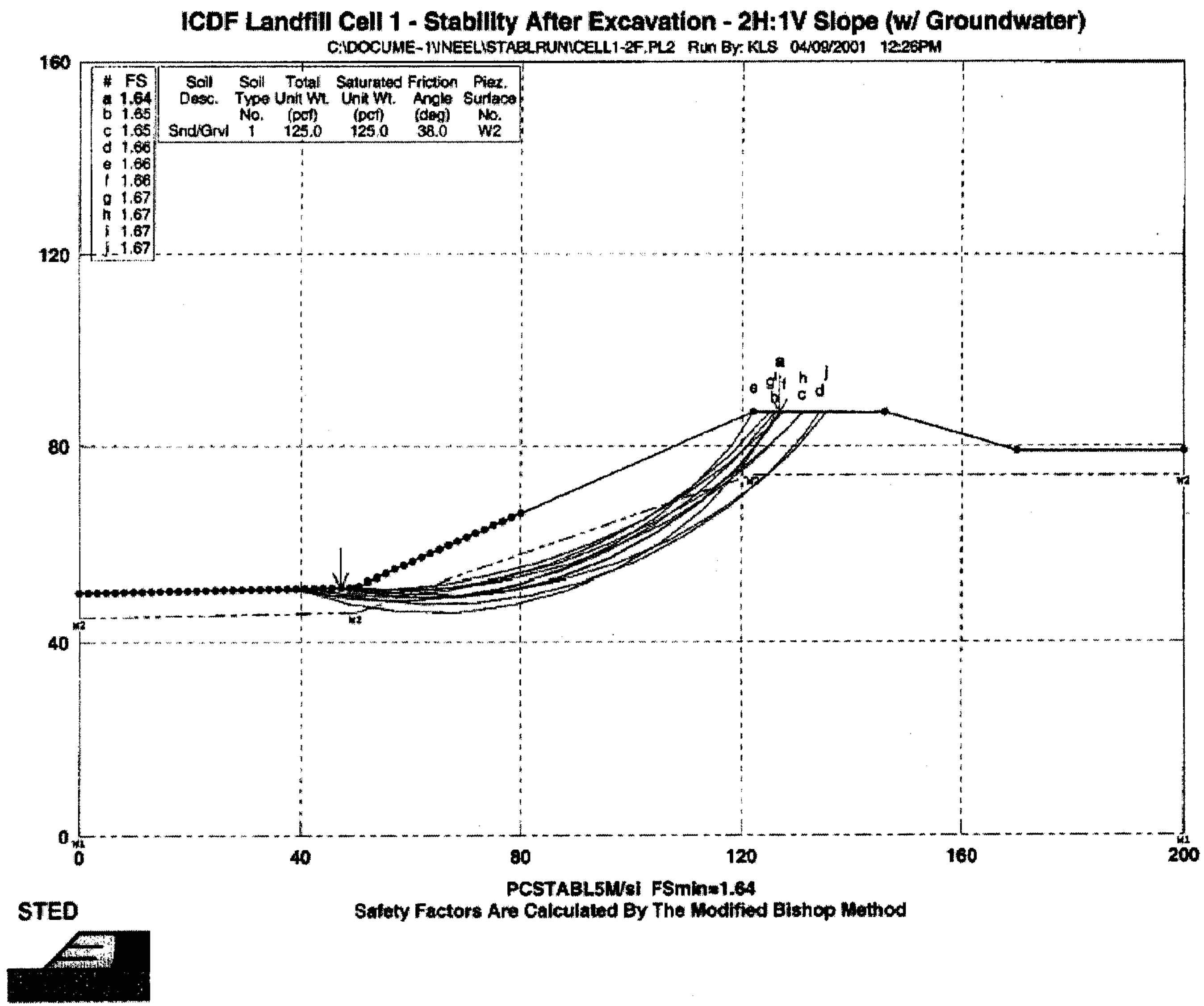

Figure 2-2. Plot of stability after excavation (with groundwater).

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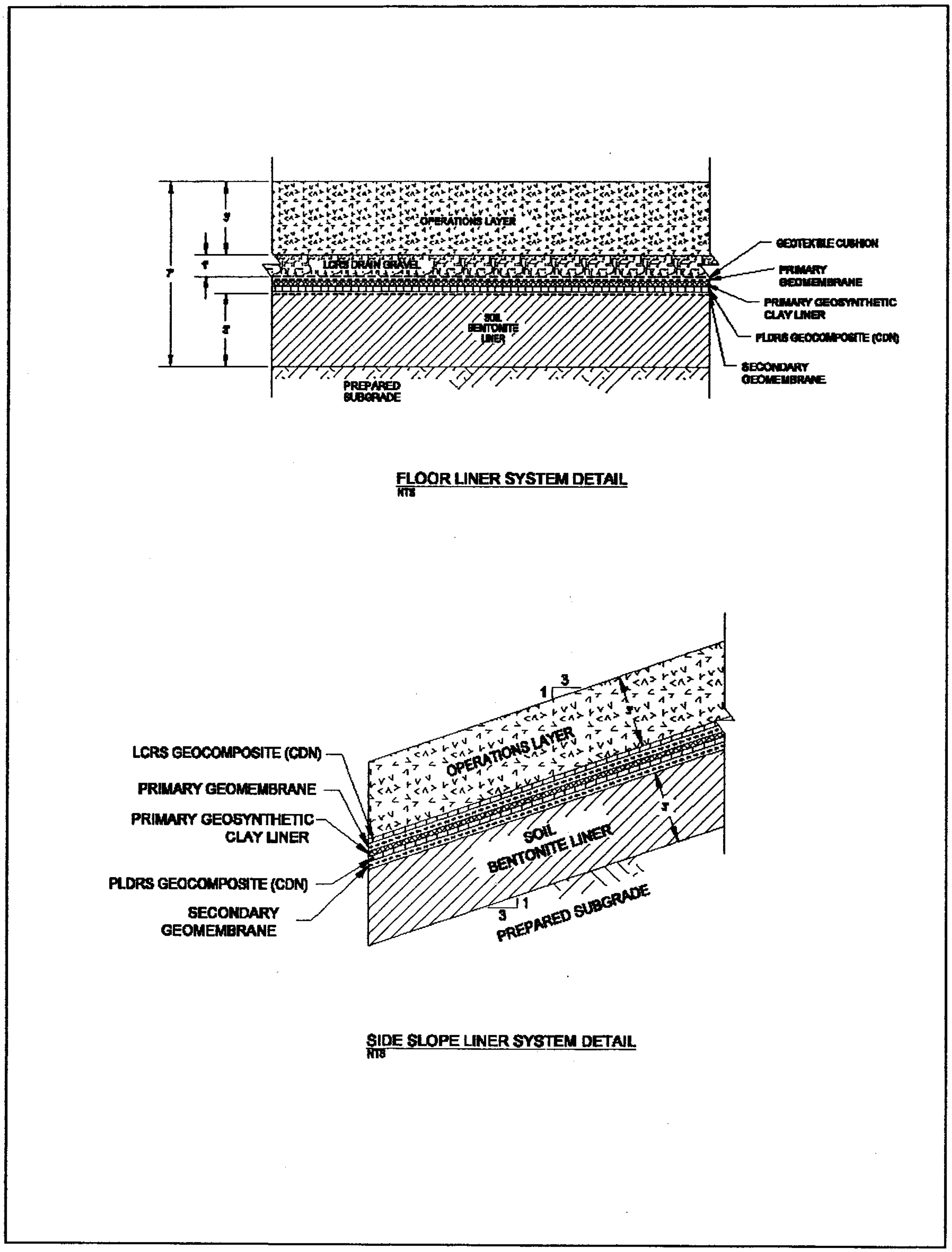

Figure 3-1. Liner systems detail.

For veneer stability of the lining system, strength values based on test data conducted under low normal stresses were considered appropriate. For this project, low normal stress in the context of veneer stability, was limited to stress levels less than 600 pounds per square foot (psf), or an equivalent of up to about 5 ft of soil. Interface shear strength data applicable to this stress level were then modeled using linear regression. In the regression analysis, the interface shear strength was represented by an effective friction angle by forcing the cohesion intercept to zero and choosing the best fit line through the data. The idea of using the effective friction angle to represent the shear strength of the interface at low normal stress is to maintain the magnitude of the shear strength while eliminating the dependency on the cohesion intercept in the strength parameter determination. For low effective confining pressures, this approach allows the shear strength to approach zero as confinement goes to zero. The regression analyses graphs are included in Appendix A and indicate a wide range of scatter in the data. This would be expected given the variability in products and test procedures inherent in this data. Primarily for this reason, site-specific interface strength testing was performed on the same materials proposed for the ICDF lining system to verify strength values determined from the regression analysis.

Appendix A contains the database of interface shear strength tests that were analyzed. Material interfaces in which test data have been analyzed under this task include soil/CDN, textured HDPE/CDN, textured HDPE/GCL, and GCL/CDN interface. Based on the measured and reported interface strength data, peak, and residual strengths of lining material interfaces were evaluated by regression analysis. For veneer stability analysis, however, residual strengths are considered to be appropriate (Stark and Poeppel 1994). For the soil/CDN interface shear strength, test data that indicate a mixture of sand and gravel (with and without silt) for the soil component, consistent with the description of the on-site native material, were evaluated in the analyses. Shear strength data for CCL/textured HDPE interface were not analyzed due to the inadequate amount of data that is available, because of the site-specific nature of CCLs. In the absence of adequate data, test results from the Cedar Hills Regional Landfill (CHRL) project (CH2M HILL 1998a; 1998b) were used in the analyses. These results indicate an interface friction angle of about 25 degrees and a cohesion of zero for the CCL/HDPE interface. Additional site-specific testing was recommended to confirm this value as discussed in "Evaluation of Geotechnical Investigations Required to Complete Design and Construction" (EDF-ER-276). Site-specific interface shear strength tests were performed after initial slope stability evaluations. Results of site-specific interface shear testing are discussed in detail in Section 5. Analyses presented herein only have been revised if lower strength values are obtained from the site-specific testing. Site-specific interface shear strength tests for the CCL/HDPE interface indicate a residual shear strength with a friction angle of 31 degrees, thus the original analyses were not revised.

Based on the above evaluations, the critical interface for the veneer stability analysis appears to be the non-woven GCL/non-woven CDN interface. Based on the regression analysis presented in Appendix A, a residual friction angle of 19 degrees was developed from the existing data for low normal stress for this interface. This value was believed to be unusually low and given the wide range of scatter in the data for this interface, more emphasis was placed on recent data provided by Montgomery Watson (1999) for products that would be similar to that used for the ICDF lining system. More recent test results provided by Montgomery Watson (1999) using exactly the same materials proposed for this project, except that the woven side of GCL was used, indicate an effective residual interface friction angle of 24 degrees. In this project, it is proposed that a non-woven side of GCL will be placed in contact with the CDN, which, as a result, could yield a higher residual friction angle than the 24 degrees that was reported. For this reason, and the fact that actual test results are available for the proposed lining material, it was decided to use a residual friction angle of 24 degrees for the GCL/CDN interface. This value matches the residual interface friction angle determined by regression analysis for the HDPE/CDN interface as the most critical interface for veneer stability. It was recommended, however, that actual interface shear strength tests be conducted for the non-woven GCL/non-woven CDN interface to confirm this value. Site-specific interface shear strength tests were performed after initial slope stability evaluations.

Results of site-specific interface shear testing are discussed in detail in Section 5. Analyses presented herein would only be revised if lower strength values are obtained from the site-specific testing. Site-specific interface shear strength tests for the GCL/CDN interface indicate a residual shear strength with a friction angle of 27.5 degrees. Site-specific interface shear strength tests for the HDPE/CDN interface indicate a residual shear strength with a friction angle of 21 degrees with a cohesion of 162 psf, which equates to a friction angle of 35 degrees with zero cohesion. Because the site-specific shear strengths for the critical interfaces exceeded the value of 24 degrees used in the original analyses, they were not revised.

The analysis for self-weight (Case 1) involved an evaluation of veneer stability under the load of the 3-ft-thick operations layer only. For equipment loads (Case 2), an equivalent equipment weight of 4,400 pounds per lineal foot of lining system such as that caused by a D6H Caterpillar bulldozer was assumed during placement of the drainage layer over the HDPE geomembrane. It was further assumed for this loading case that the seepage height would be zero. For the seepage case (Case 3), the maximum allowable head over the side slope lining system is 6 in. for stability purposes. FSs corresponding to seepage heights of 3 in. and 6 in. were evaluated. A maximum slope height of 40 ft was used in performing the analyses.

3.1.2 Results

For static analyses, the desired minimum FS is 1.5 for the long-term and 1.3 for the short-term loading conditions (see Section 1). These are industry-accepted values, and account for uncertainties in soil and interface properties, assumptions made during analysis, irregularities in actual slope geometry, and construction quality. Although veneer stability is technically not a case of long-term loading condition, a minimum FS of 1.5 was used for self-weight loading (Case 1). Equipment loading (Case 2) and seepage forces (Case 3) were treated as short-term loading (minimum FS of 1.3) due to the relatively short duration of this loading (during construction and prior to refuse filling) compared to Case 1.

The minimum calculated FS for each analysis case using the SLOPBASE program corresponds to the FS when no anchor force is required. For cases in which the FS is less than the desired minimum, the anchor force required to achieve that minimum was determined from the output of the program. The required anchored force was then compared to the yield strength of the stiffest element of the lining system, which, for this project, is the HDPE geomembrane.

Table 3-1 presents the results of the veneer stability analyses for all three loading conditions. FS values reported in Table 3-1 are reported to the nearest 0.05 and are conservatively rounded to the lower value. For example, for Case 1 the FS was calculated as between 1.45 and 1.50 but reported as 1.45. The results of the analyses for Cases 2, 3a, and 3b indicate that no anchor force is required to achieve the required minimum FS for slope stability, as long as (1) the seepage height in the drainage layer on side slopes is no higher than 6 in., and (2) the bulldozer does not operate directly on the side slope operations layer if seepage height builds up in the operations layer. For condition (1), it should be noted that the results of Hydraulic Evaluation of Landfill Performance (HELP) modeling (details presented in "Leachate Generation Study" [EDF-ER-269]) indicate that there is negligible seepage height on the lining system side slope. Condition (2) is most likely immediately following a rainfall. Additional calculations were performed to better quantify this restriction. These calculations are presented at the end of Appendix A. The calculations were based on the rainfall intensity and the capacity of the side slope drainage system. The evaluation determined that equipment (with ground pressure less than 4,400 pounds per lineal foot) may operate on the side slopes until a rain event in excess of 0.15 in. per hour occurs. In that event, equipment should be kept off of the side slope (directly on the operations layer) and should not be permitted to operate on slopes until one hour after the end of the rainfall event.

Table 3-1. Summary of veneer stability analysis for ICDF landfill[a].

Loading Condition	Slope Height (ft)	Equipment Load (lb/ft)	Seepage Height (ft)	FS	Required Tension to Achieve Minimum FS (lb/ft)
Case 1 (Dead Load)	40	0	0	1.45	440
Case 2 (Dead Load + Equipment Load)	40	4,400	0	1.30	0
Case 3a (Dead Load + 3-in. Seepage Forces)	40	0	0.25	1.35	0
Case 3b (Dead Load + 6-in. Seepage Forces)	40	0	0.50	1.30	0

a. The following constants were assumed for this analysis:
Cover soil thickness = 36 in.
Cover soil unit weight = 120 pcf
Critical interface shear strength: δ24 degrees, c = 0 (textured HDPE/CDN and non-woven GCL/CDN interfaces)

As shown in Table 3-1, a portion of the tensile strength of the HDPE will be required to achieve the minimum FS of 1.5 against self-weight loading (Case 1). This required tension force (440 lb/ft) constitutes only 29% of the yield strength of the HDPE geomembrane (1,500 lb/ft), which is acceptable. Calculations for the anchor trench design are presented in "Liner and Final Cover Long Term Performance Evaluation and Final Cover Life Cycle Expectation" (EDF-ER-281). Because of the differences in stiffness of the lining system, it has to be noted that a portion of the load will be transferred to the HDPE geomembrane only after the shear resistance of the critical interface in the lining system has been fully mobilized and sufficient movement has occurred. Full mobilization of shear resistance at the critical interface will only occur when the FS against shear failure in that interface approaches 1.0. Given that the calculated FS is about 1.45, it is highly unlikely that full mobilization of shear resistance at the critical interface will occur under static loading conditions, thus limiting the actual tension force on the HDPE geomembrane.

It is important that only equipment with a weight-to-width ratio less than or equal to that of a D6H bulldozer (4,400 lb/ft) be used for liner construction and maintenance on the side slopes when operating directly on the operations layer. Additionally, because it was assumed that no seepage occurs within the drainage layer while equipment is operating on the slope, equipment may operate on the side slopes until a rain event in excess of 0.15 in. per hour occurs. In that event, equipment should be kept off of the side slope (directly on the operations layer) and should not be permitted to operate on slopes until one hour after the end of the rainfall event.

Results of the seismic evaluation indicate that the seismic coefficient required to achieve a minimum FS of 1.1 is 0.1. This seismic coefficient is equivalent to a peak-ground acceleration of 0.2g as detailed in Section 1. Appendix A contains the printouts of the output from the program SLOPBASE for all loading cases considered in the veneer stability analysis of the landfill lining system.

3.2 Evaporation Pond

The approved alternative lining system for the evaporation pond consists of the following:

- Two 60-mil HDPE geomembrane layers (sacrificial and primary)
- An internally reinforced primary GCL layer
- A 3-ft sand/gravel drainage layer to serve a dual purpose as a leak detection drain layer and freeze/thaw protection for underlying GCL
- A cushion geotextile (12-oz nonwoven filter fabric)
- A 60-mil secondary HDPE geomembrane layer
- An internally reinforced secondary GCL layer
- A 1-ft-thick base soil layer consisting of natural clay from the Rye Grass Flats Borrow Area
- Prepared subgrade.

Evaluation of shear strength parameters for the material interfaces of the above lining system indicated that the critical interface is between the non-woven cushion geotextile and the textured HDPE geomembrane. This interface will have similar interface strength as the HDPE/CDN interface (δ24 degrees, c= 0), as discussed in Section 3.1.1.

Veneer stability analysis was then conducted using the critical interface discussed above. As in Cell 1, the veneer stability analysis was conducted using the program SLOPBASE and using the three loading cases mentioned above. A maximum slope height of 10 ft was used in the calculations. Seismic evaluation was also included, but was only limited to determining the required acceleration to achieve the desired minimum FS, as was done for Cell 1.

Table 3-2 shows the results of the veneer stability analysis for evaporation pond. These results indicate that the alternative lining system proposed for the evaporation pond satisfies the minimum FS requirements stated in Section 1 for the expected range of loading conditions. FSs are greater than the minimum 1.5 for long-term loading conditions for all cases except under equipment loading. However, the evaporation pond lining system will only be subject to equipment loads during construction (during placement of the drain gravel and operations layer). For this short-term loading condition a minimum FS of 1.3 is acceptable.

Results of the seismic evaluation indicate that the seismic coefficient required to achieve a minimum FS of 1.1 is 0.2. This seismic coefficient is equivalent to a peak-ground acceleration of 0.4g as detailed in Section 1. Appendix A contains the printouts of the output from the program SLOPBASE for all loading cases considered in the veneer stability analysis of the evaporation pond lining system.

Table 3-2. Summary of Veneer Stability Analysis for Evaporation Pond[a].

Loading Condition	Slope Height (ft)	Equipment Load (lb/ft)	Seepage Height (ft)	Factor of Safety (FS)	Required Tension to Achieve Minimum FS (lb/ft)
Case 1 (Dead Load)	10	0	0	1.80	0
Case 2 (Dead Load + Equipment Load)	10	4,400	0	1.30	0
Case 3a (Dead Load + 6-in. Seepage Forces)	10	0	0.25	1.75	0
Case 3b (Dead Load + 12-in. Seepage Forces)	10	0	0.50	1.60	0

a. The following constants were assumed for this analysis:
Cover soil thickness = 36 in.
Cover soil unit weight = 120 pcf
Critical interface shear strength: $\delta = 24$ degrees, $c = 0$ (textured HDPE/NW cushion geotextile)

4. GLOBAL STABILITY ANALYSES

Global stability analyses involve evaluation of the potential for sliding of the waste mass along the critical interface after the refuse is placed (interim fill stability) and after the final landfill cover has been installed (final configuration stability). This analysis also included evaluation of the potential global failure of the evaporation ponds. Figure 4-1 shows the plan view of the completed ICDF landfill. Global stability analysis was evaluated using Section B-B, which cuts north-south through the highest point of the landfill and the steepest portion of the landfill bottom.

Both static and seismic loading conditions were included in the global stability analyses. The analysis for seismic loading, however, was only limited to determining the acceleration levels at which the FS against global stability equals 1.3. Discussion of return periods associated with these acceleration levels and implications of this seismic event for design of the ICDF landfill and evaporation pond are given in "Seismic Evaluation of Landfill and Evaporation Pond" (EDF-ER-282).

The computer program PCSTABL5 (FHWA 1988) was used to model the landfill and to evaluate the effects of earthquake loading as discussed in Section 1. The required input to the program included the different material properties, interface shear strengths, and earthquake forces. These input parameters are discussed in the following sections.

4.1 Interim Filling Stability

Interim fill stability involves stability evaluation during operation of the landfill. Two cases were evaluated under this analysis (as shown in Figure 4-2):

- Case A involved a stability evaluation for the case when Cell 1 has reached final capacity and excavation for Cell 2 commences. This evaluation was aimed at assessing the FS against sliding of the waste mass in the direction of Cell 2 due to removal of the perimeter berm between the two cells. The removal of the perimeter berm represents a critical case scenario in that the FS is reduced due to the loss of the buttressing effect from the berm. The analysis conservatively assumed a worst-case scenario, that waste placement in Cell 1 would reach full height (as shown in Figure 4-2) prior to Cell 2 excavation and construction. It is anticipated that Cell 2 construction will occur well in advance of Cell 1 achieving full capacity. For the purpose of analysis, the external side slope of the waste material in Cell 1 was assumed to be 2H:1V.

- Case B involved stability evaluations for Cell 1 at various stages of waste filling. This case was evaluated to establish criteria for placement of the refuse and the limiting amount of refuse that can be placed without failure. In this analysis, PCSTABL runs were made by varying the length (lateral extent) of the refuse and then calculating the FS for each length.

Because of the similarity in the geometry for Cell 1 and Cell 2, the analysis results considering an individual cell for Cell 1 (Case A and Case B) are also applicable to that of an individual cell analysis for Cell 2. For the case where both Cell 1 and Cell 2 are filled, the "bowl" effect of the landfill configuration will buttress the waste mass against sliding, which results in a higher FS than for the case of individual cell analyses.

The components of the lining system in the landfill floor are similar to the ones described in Section 3 above for the side slope. The only difference in the lining configuration between the floor and the side slope is that a layer of gravel sandwiched between two geotextiles replaces the upper CDN drainage layer that is used in the side slopes, as shown in Figure 3-1.

4.1.1 Material Properties and Critical Surfaces

The section modeled in the PCSTABL analyses for Cell 1 is similar to the one shown in Figure 4-2. A seepage height of 12 in. is assumed at the bottom and on the side slopes to conservatively model the maximum leachate head permitted on the lining system. HELP modeling has indicated that actual head buildup will be much lower than 12 in.

Four groups of material were represented in the global stability models: (1) the refuse, (2) the layers of material on the bottom of the landfill, (3) the layers of material on the side slopes, and (4) the native subgrade. Strength parameters and material properties required for input include interface cohesion and friction angle, and moist and saturated unit weights. The values used for this project are summarized in Table 4-1. Strength values for the native subgrade were developed from the subsurface information given in the geotechnical report (DOE-ID 2000). The development of strength parameters for the lining system interfaces is discussed in more detail below.

4.1.1.1 Strength Properties. For a realistic assessment of FS against global stability of landfills, the current state-of-the-practice is to use residual strengths on the side slopes and peak strengths for the landfill bottom (Stark and Poeppel 1994). This is because shear displacements are relatively small at the base of the landfill, allowing mobilization of peak strengths. It is considered prudent, however, to verify that the FS is greater than 1.0 (i.e., 1.1) if residual strengths are used at the bottom of the landfill. Thus, both the peak and residual strengths were used to evaluate static global stability in this report. For the case of global stability under seismic loading, peak strengths were used.

As the final buildout height of the landfill will be on the order of 40 ft above the bottom, the normal pressure on the bottom liner will be on the order of 4,500 psf. Therefore, interface strength parameters appropriate for these high normal stress conditions were selected. As discussed in Section 3 of this report, the interface strength parameters were estimated from published data in the literature, from test data provided by manufacturers of landfill lining components, and from other projects involving similar materials (STS 1993; CETCO 1994; TRI 1993; Bentofix 1994; CH2M HILL 1993; GeoSyntec 1994a and b; AGP 1995; GeoSyntec 1996a and b; CETCO 1996; GeoSyntec 1997; Emcon 1993; Clem Corp 1994; CH2M HILL 1994; CETCO 1995; and CH2M HILL 1998a). Appendix B of this report contains the interface shear strength data that were analyzed for global stability.

The interface shear strength data were divided into two groups according to the normal stress level applied during the test to simulate vertical loading of the lining system. Applied normal stresses of 2,000 psf or lower are designated "low stress level" and applied normal stresses higher than 2,000 psf are designated "high stress level." The value of 2,000 psf was used based on the findings and test results by Byrne (1994), which indicate that internal shear strength of GCL is the critical component of the lining system at normal pressures of greater than 2,000 psf.

Interface strength parameters estimated under low stress level tests are appropriate for use in the upper portion of the landfill side slope while parameters estimated from high stress level tests are appropriate for use in the bottom portion of the side slope and at the landfill base. The distinction between the "upper" and "lower" portions of the landfill side slope is determined by estimating the depth below the maximum elevation of the waste at which the overburden pressure equals 2,000 psf. This depth is calculated to be about 18 ft for the ICDF landfill (115 pcf x 18 ft = 2,070 psf).

4.1.1.2 Bottom Lining System—Critical Interface. For the bottom liner system, the critical interface based on peak strength data is either between the textured HDPE geomembrane and CCL interface or between the textured HDPE geomembrane and GCL. Regression analysis of published data

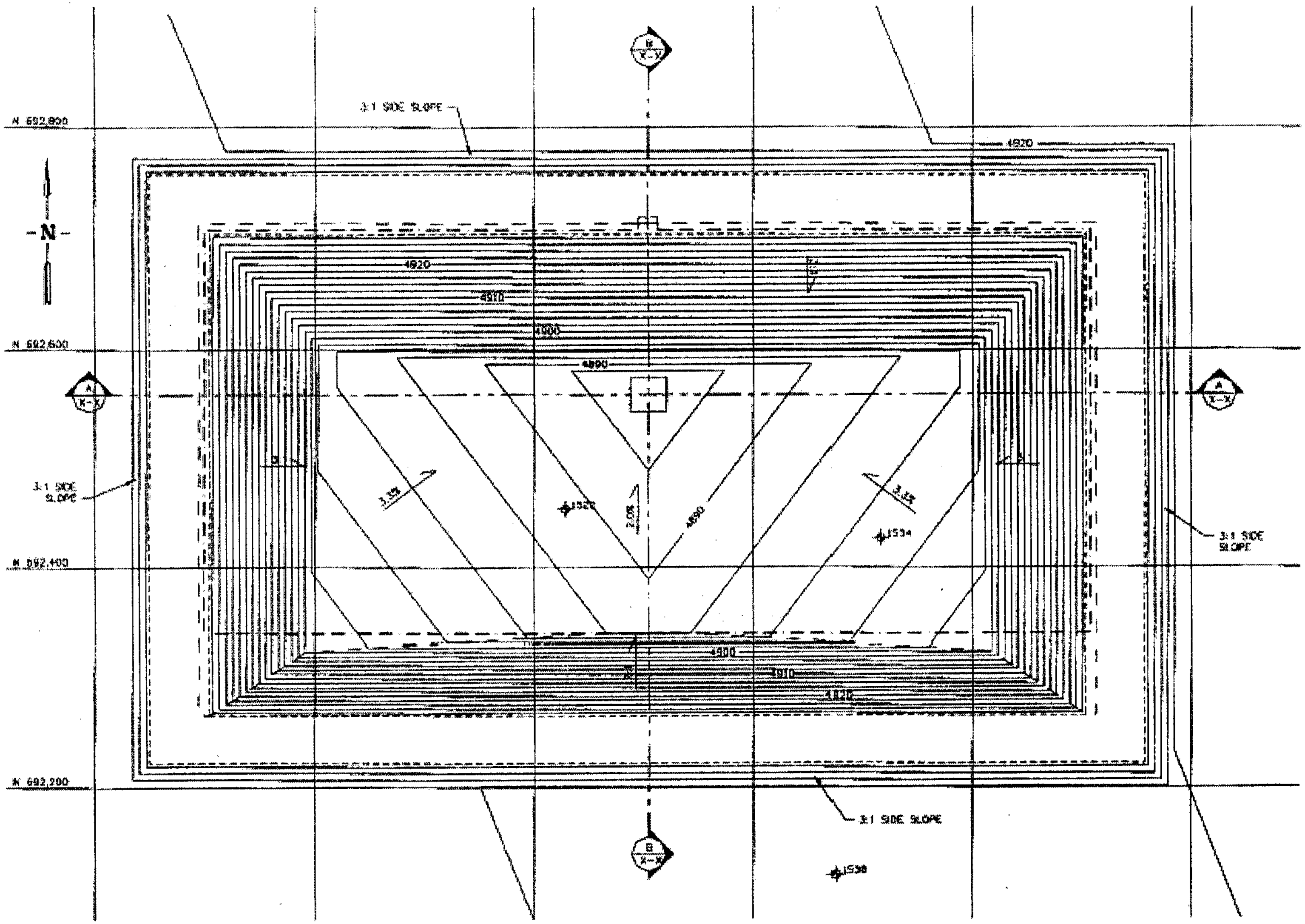

Figure 4-1. Completed ICDF (Cell 1) landfill plan.

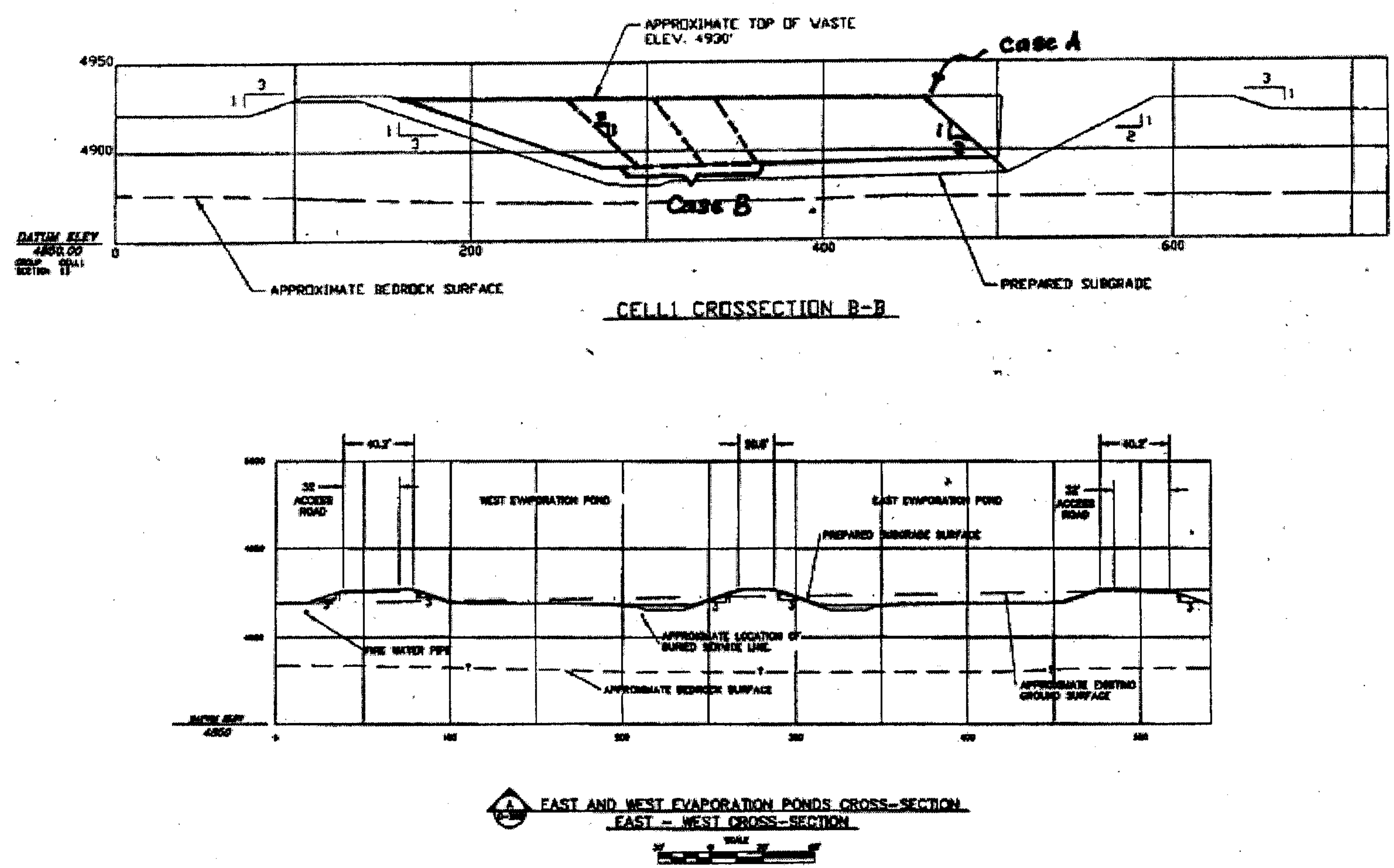

Figure 4-2. Typical cross-sections for Cell 1 and evaporation ponds.

and test results for the latter interface (see Appendix B) indicates a peak interface shear strength of 574 psf (cohesion) and 18 degrees (friction angle). This interface strength is slightly greater than that determined for the HDPE/CCL interface based on the CHRL test data (CH2M HILL 1998a) for high normal stress. Test data from this project reported a peak HDPE/CCL interface strength of 524 psf (cohesion) and 18 degrees (friction angle). On the basis of residual strength, however, the internal strength of the GCL governs. Results of regression analysis of internal residual shear strength data for GCL (see Appendix B) indicate a residual cohesion of 349 psf and a residual friction angle of 6 degrees. Additional site-specific testing was recommended to confirm these values as discussed in "Evaluation of Geotechnical Investigations and Calculations Required to Complete Design and Construction" (EDF-ER-276). Site-specific interface shear strength tests were performed after initial slope stability evaluations. Results of site-specific interface shear testing are discussed in detail in Section 5. Analyses presented herein would only be revised if lower strength values are obtained from the site-specific testing. Site-specific interface shear strength tests for the HDPE/CCL interface indicate a peak shear strength with a friction angle of 29.5 degrees and a cohesion of 13 psf. Site-specific interface shear strength tests for the GCL internal shear strength indicate a residual shear strength with a friction angle of 17 degrees and a cohesion of 404 psf. Because the site-specific shear strengths for the critical bottom lining interfaces exceeded the values used in the original analyses, they were not revised.

4.1.1.3 Side Slope Lining System—Critical Surface. For the side slope lining system, the critical interface in the upper portion of the side slope liner is the interface between the CDN and GCL (interface 1); and the critical surface in the lower portion of the side slope liner is the internal strength of the GCL (interface 2). Results of regression analysis of published test data for the CDN/GCL interface, as shown in Appendix B, indicate a residual cohesion of 19 psf and a residual friction angle of 15 degrees. For the internal strength of GCL, the internal residual strength based on linear regression was 349 psf (cohesion) and 6 degrees (friction angle).

Site-specific interface shear strength tests for the GCL/CDN interface indicate a residual shear strength with a friction angle of 22 degrees and a cohesion of 39 psf. Site-specific interface shear strength tests for the GCL internal shear strength indicate a residual shear strength with a friction angle of 17 degrees and a cohesion of 404 psf. Because the site-specific shear strengths for the critical side slope lining interfaces exceeded the values used in the original analyses they were not revised.

Table 4-1 summarizes the material and strength parameters used in the global stability analyses.

Table 4-1. Summary of material properties and strength parameters used in global stability.

		Interface Peak Strength		Residual Strength for Interfaces		
Material Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Comments and Sources of Information
Refuse/Waste	110	0	26	0	26	Typical value for silt and sand deposited under very loose condition (NAVFAC 1986).
Landfill Bottom Interface	120	524	18	349	6	Critical interface: (a) HDPE over CCL for peak; (b) GCL internal strength for residual. See discussion above.

Table 4-1. (continued).

Material Type	Unit Weight (pcf)	Interface Peak Strength: Cohesion (psf)	Interface Peak Strength: Friction Angle (degrees)	Residual Strength for Interfaces: Cohesion (psf)	Residual Strength for Interfaces: Friction Angle (degrees)	Comments and Sources of Information
Side Slope Interface (< 2,000 psf)	120	N/A	N/A	19	15	Critical surface at upper portion of side slope: CDN over GCL. See discussion above.
Side Slope Interface (> 2,000 psf)	120	N/A	N/A	349	6	Critical interface at bottom portion of side slope: Internal GCL. See discussion above.
Native Subgrade	125	0	38	0	38	Geotechnical Report for Conceptual Design (DOE-ID 2000).

4.1.2 Results

The results of global stability analysis are given in Tables 4-2 and 4-3 for Case A and Case B, respectively. Computer printouts of the results are attached in Appendix B of this report.

Results for Case A indicate that the FS against global failure by sliding along the critical interface is more than acceptable even if the waste placement in Cell 1 reaches full height prior to the start of Cell 2 excavation. The lowest seismic coefficient that will be required to reduce the FS to the minimum acceptable value is approximately 0.11. This seismic coefficient is equivalent to a peak-ground acceleration of 0.22g as detailed in Section 1.

Results for Case B suggest that, to maintain a minimum FS of 1.3 (for temporary condition, static loading) against shear failure in the lining system during placement of the waste, the waste should be placed in lifts and spread over a distance of not less than 100 ft, as measured from the toe of the landfill to the outer limit of the waste.

Table 4-2. Results of global stability analysis (interim filling, completed Cell 1—Case A)[a].

Interface Strength @ Bottom Liner	Static FS	Seismic Coefficient (k) to achieve FS = 1.3
Peak	10.17	0.28
Residual	4.66	0.11

a. Precision of FS calculations given for comparative purposes only.

Table 4-3. Results of global stability analysis (interim filling, filling Cell 1—Case B).

Length[a] (ft)	Interface Strength @ Bottom Liner	FS[b] (Static)
50	Peak	1.13
50	Residual	0.94
75	Peak	1.48
75	Residual	1.06
100	Peak	2.02
100	Residual	1.30

a. Defined as the distance from the toe of the landfill slope to the toe of the placed refuse.

b. Precision of FS calculations given for comparative purposes only.

4.2 Final Configuration Stability

This analysis involves evaluation of the potential for sliding of the refuse mass after the final cover has been installed. Figure 4-3 shows the typical configuration that was analyzed under this loading condition. PCSTABL5 was used to search for failure surfaces that could propagate from the toe of the cover soil and into the refuse. Both circular and sliding block failure surfaces were specified. A simplified subsurface profile was used to represent the multiple layers of materials that are proposed as part of the final cover design. This simplification is expected to yield conservative results in that the strengths of the stronger materials were neglected and were assigned strengths similar to the native soils at the site.

Results of this analysis indicate that the final landfill configuration after the cover has been installed will have adequate capacity under both static and seismic loading. The estimated FS for a sliding block failure within the cover soil is at least 7.4 under static loading. Under seismic loading, the seismic coefficient required to achieve a FS of 1.3 (for long-term loading) is about 0.45. This seismic coefficient is equivalent to a peak-ground acceleration of 0.90g as detailed in Section 1. Interpretation of the results for seismic stability is given in "Seismic Evaluation of Landfill and Evaporation Pond" (EDF-ER-282). Computer printouts for this analysis are attached in Appendix B.

Long-term veneer stability of the final cover 2.5H:1V perimeter slopes (as shown in Figure 4-3) was also evaluated. The perimeter final slopes consist of 2 ft of riprap armor over 3 ft of filter media. The analysis was simplified by assuming one material property for the full 5-ft-perimeter cover slope thickness.

A soil friction angle of 38 degrees was conservatively selected of the cover material. This strength value is based on the strength determined for native granular material as discussed in Section 2. Native granular material is likely to compromise the filter media, however the riprap may have to be imported and has much higher shear strength than the native material.

Results of this analysis indicate that the final cover perimeter slopes are stable under both static and seismic loading. The estimated FS for a sliding block failure within the cover soil is at least 2.2 under static loading. If the cover soil on the perimeter slopes was to become saturated to 50% of its thickness (2.5 ft), a conservative assumption given the arid nature of the site and the free-draining nature of the material, the FS would only be reduced to 1.80. Under seismic loading, the seismic coefficient required to achieve a FS of 1.3 (for long-term loading) is about 0.30. This seismic coefficient is equivalent to a peak-ground acceleration of 0.60g as detailed in Section 1. Interpretation of the results for seismic stability is given in "Seismic Evaluation of Landfill and Evaporation Pond" (EDF-ER-282). Computer printouts for these analyses are included in the veneer stability calculations in Appendix A.

4.3 Evaporation Pond

Stability analyses were conducted to evaluate the potential for global failure of the evaporation ponds. Global failure, in this context, refers to a deep-seated failure that includes inboard portions of the evaporation pond. Both circular and sliding block failure surfaces were specified in the analyses. Since the analyses primarily involve long-term loading conditions, minimum FSs of 1.5 and 1.3 (see Section 1) are considered appropriate for static and seismic loading, respectively.

Stability analyses were conducted using a similar cross section of the evaporation ponds as that shown in Figure 4-2. Results of stability analyses indicate that the minimum FS for global failure of the evaporation ponds under static loading is 3.9, which is higher than the minimum required FS of 1.5. Under long-term seismic loading, the seismic coefficient needed to drop the FS to about 1.3 is 0.37. This seismic coefficient is equivalent to a peak-ground acceleration of 0.74g as detailed in Section 1. Appendix B contains the printouts of PCSTABL runs for these loading cases.

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5. SITE-SPECIFIC INTERFACE SHEAR STRENGTH TESTING

Slope stability analyses presented herein were performed using interface strength characterization data for the geosynthetics from previous projects, the technical literature, and manufacturers. This data, along with linear regression plots of the data to determine appropriate shear strength values under various loading conditions, is presented in Appendices A and B. Variability in test set-up and products used in this data required that interface shear strength testing on site-specific lining system products be performed to verify the strength parameters used for lining system interfaces.

Interface shear strength testing was performed by Precision Geosynthetic Laboratories in Anaheim, California. As detailed in "Evaluation of Geotechnical Investigations and Calculations Required to Complete Design and Construction" (EDF-ER-276) interface tests were performed on the following interfaces:

- Operations Layer Soil/CDN
- CDN/Textured HDPE Geomembrane
- Textured HDPE Geomembrane/GCL
- GCL/CDN
- GCL Internal Shear
- Textured HDPE Geomembrane/Soil-Bentonite Admix.

Testing was performed on all lining system interfaces in June 2001 and then again in March 2002 for the textured HDPE geomembrane interfaces. The additional testing was necessitated by a change in textured geomembrane product selection due to the manufacturer's discontinuation of the original product.

Materials used for the testing were the same materials proposed for the actual lining system construction as follows:

- Operations Layer—Native alluvium material excavated from test pits at the ICDF site
- CDN—FabriNet geocomposite supplied by GSE Lining Systems
- Textured HDPE Geomembrane— HD Textured (60-mil) supplied by GSE Lining Systems
- GCL—Bentomat DN (double non-woven geotextile) supplied by CETCO
- Soil-Bentonite Admix—Same material used for Soil Amendment Study laboratory testing.

All interface shear tests were conducted in accordance with ASTM D5321 and ASTM D6243 (for interfaces with GCL). All interfaces, except for the GCL internal shear, were tested at both high and low normal stresses. The GCL internal shear was tested at high normal stress only as the internal shear strength of reinforced GCLs (such as Bentomat DN) is only relevant under high normal loads as discussed in Section 4.1.1.1. Low normal stress tests were performed at loads of 100, 200, and 500 psf. High normal stress tests were performed at loads of 1,000, 4,000, and 8,000 psf. Other specific test

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Table 5-2. Summary of high normal stress site-specific interface shear tests.

Lining System Interface	Test Configuration No.[a]	Peak Friction Angle (Degrees)	Peak Cohesion (psf)	Residual Friction Angle (Degrees)	Residual Cohesion (psf)
Ops Layer/ CDN	2	32.0	212	27.4	385
CDN/HDPE	4R[b]	26.1	258.5	12.9	214.3
HDPE/GCL	6R[b]	26.4	525.9	12.6	548.6
GCL/CDN	8	25.1	82	22.2	39
Internal GCL	9	13.9	1207	17.2	404
HDPE/CCL	11R[b]	29.5	13	28.6	20.9

a. See Appendix C for test data for each configuration number.
b. "R" designates revised test for replacement textured HDPE geomembrane (HD textured).

Data from site-specific testing was compared to the strength values used in the global stability analyses. Site-specific data was added to the shear strength database used to determine the strength values for the original global stability analysis (see Appendix B). With the addition of the site-specific data, the linear regression plots were recalculated for each interface and are presented in Appendix C. In all but one case the strength value determined with the site-specific data increased, indicative that the site-specific test values were equal to or greater to those used in the original global stability analyses. The HDPE/CDN interface indicated a slight decrease in residual shear strength with the addition of site-specific data. However, this is mitigated for two reasons; 1) the decrease is only minimal from a friction angle of 15 degrees with a cohesion of 129 psf to a friction angle of 14 degrees with a cohesion of 146 psf; and 2) the HDPE/CDN interface was not the critical interface used in the analysis and the revised strength value still exceeds that of the critical interface strengths used in the analyses.

Particular emphasis was placed on those interfaces deemed critical to global stability and a direct comparison of the values used in the original analyses and the site-specific results was provided previously in Section 4.1.1.2 for the bottom lining system critical interface and Section 4.1.1.3 for the side slope lining system critical interface.

Results of site-specific interface shear testing indicate that shear strengths used for the global stability analyses are less than or equal to those obtained from the site-specific testing. Site-specific testing confirmed that strength values used in the original analysis are adequate for landfill and evaporation pond global stability. Thus, the analyses presented in the original global slope stability evaluations were not revised.

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